2. Evaluation Overview

2.1 Scope

This section provides a discussion of the history of the development of steel moment-frame buildings and the general earthquake damage and vulnerabilities associated with such buildings. An overview of the evaluation procedures contained in these recommended criteria is presented along with corresponding sections regarding material property and condition assessment approaches.

2.2 Steel Moment-Frame Building Construction

2.2.1 Introduction

Steel frames have been used in building construction for more than one hundred years. In the early 20th century, typical steel frames were of riveted construction. Beam-column connections were of two common types illustrated in Figure 2-1, in which beams were connected to columns using either stiffened or unstiffened angles at the top and bottom beam flanges. Designers often assumed that these assemblies acted as “pinned” connections for gravity loads and that the stiffened connections would act as “fixed” connections for lateral loads. Although some hot-rolled shapes were available, these were typically limited to beam applications. Columns and girders were often fabricated out of plate and angle sections. Frames were typically designed for lateral wind loading, employing approximate methods of frame analysis, such as the portal method or cantilever method.

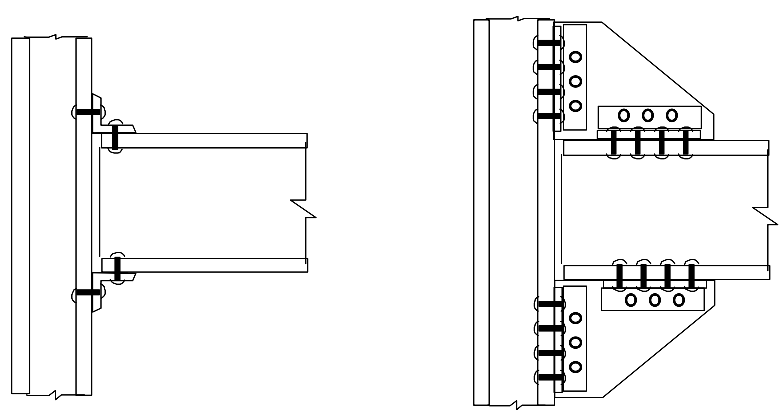

Figure 2-1 Typical Early Beam-Column Connections

Most early steel frame buildings had exterior walls of unreinforced masonry. The exterior building frame was typically embedded in these walls providing for significant interaction between the steel and masonry elements. Although these buildings were usually designed neglecting the effects of the masonry in lateral load resistance, in actuality there is significant interaction between the masonry walls and steel frames and the masonry provides much of the lateral resistance of such buildings.

Infilled masonry construction remained common until the early 1940s. At about that time, reinforced, cast-in-place concrete walls began to replace the masonry used in earlier buildings. These reinforced concrete walls were typically designed to provide the lateral resistance for the structure, and the steel frame was often designed only to carry gravity loading, though some buildings with a "dual" system of concrete walls and steel moment frames also were constructed during this period. Steel moment frames without infill walls came into wider use when curtain wall systems became popular, in the late 1940s and early 1950s. This was the time when moment resistance and stiffness of the connections became a critical issue. The earliest steel moment frames employed riveted or bolted connections similar to those used in the earliest infill masonry buildings. However, as design procedures became more sophisticated and the building codes began to require design for larger seismic forces, designers started to design fully restrained connections intended to develop the full flexural strength of the beams. Connections were usually complex and expensive, consisting, for example, of plates, stiffened angles, and T-sections that were riveted or bolted.

During the Second World War, structural welding was introduced in the ship-building industry as a means of speeding ship construction. It is interesting to note that these early attempts at welded construction were not entirely successful and were plagued by unanticipated fracture problems. Several Liberty Ships, a class of cargo vessel, some of which were among the first to employ welded hull construction, experienced massive fracture damage and a few actually fractured in two and sank. These problems were eventually traced to sharp corners at openings in the hull and superstructure as well as to inadequate notch toughness in the materials of construction. By the 1950s, however, these problems were largely mitigated by improved design and construction practice and welded construction had completely replaced the earlier bolted and riveted construction techniques formerly prevalent in this industry.

In the late 1950s, structural welding began to spread to the building industry. This trend, together with the need to design strong and stiff, but economical, connections, accelerated a design shift from riveted or bolted, partially restrained connections to designs employing welded, fully restrained connections. Many different types of welded connections were used, the earlier ones consisting mostly of shop-welded, field-bolted cover plates connecting the beam flanges to the columns. In the late 1950s the field-welded direct connection between beam flanges and column flanges started to see some use. Experimental research performed in the mid to late 1950s, primarily at Lehigh University, provided criteria for welding and for continuity plate requirements to minimize web crippling and column flange distortions. Additional experimental research performed in the mid 1960s to early 1970s at the University of California at Berkeley provided evidence that certain types of butt-welded beam-flange-to-column-flange connections could behave satisfactorily under cyclic loading. These data lead to widespread adoption of the bolted-web, welded-flange, beam-column connection shown in Figure 2-2, by engineers designing for earthquake resistance.

2.2.2 Welded Steel Moment-Frame (WSMF) Construction

Today, WSMF construction is commonly used throughout the United States and the world, particularly for mid-rise and high-rise construction. Prior to the 1994 Northridge earthquake, this type of construction was considered one of the most seismic-resistant structural systems, due to the fact that severe damage to such structures had rarely been reported in past earthquakes and there was no record of earthquake-induced collapse of such buildings, constructed in accordance with contemporary US practice. However, the widespread reports of structural damage to such structures following the Northridge earthquake called for re-examination of this premise.

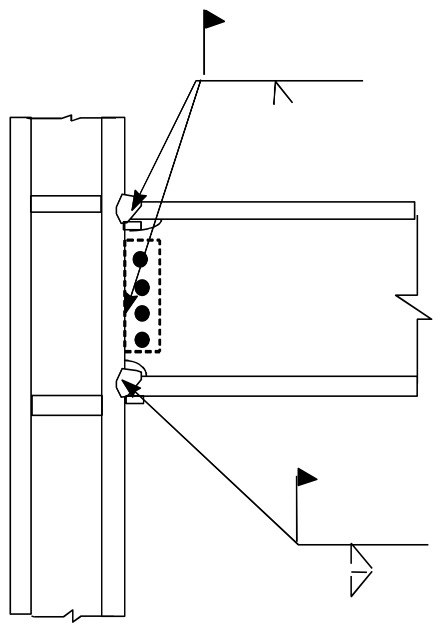

Figure 2-2 Typical Bolted Web, Welded Flange Connection

Steel moment-frame buildings are designed to resist earthquake ground shaking, based on the assumption that they are capable of extensive yielding and plastic deformation, without loss of strength. The intended plastic deformation consists of plastic rotations developing within the beams, at their connections to the columns, and is theoretically capable of resulting in benign dissipation of the earthquake energy delivered to the building. Damage is expected to consist of moderate yielding and localized buckling of the steel elements, but not brittle fractures. Based on this presumed behavior, building codes permit design of steel moment-frame structures for lateral forces that are approximately 1/8 those which would be required for the structure to remain fully elastic. Supplemental provisions within the building code, intended to control the amount of interstory drift sustained by these flexible frame buildings, typically result in structures which are substantially stronger than this minimum requirement and in zones of moderate seismicity, substantial overstrength may be present to accommodate wind and gravity load design conditions. In zones of high seismicity, most such structures designed to minimum code criteria will not start to exhibit plastic behavior until ground motions are experienced that are 1/3 to 1/2 the severity anticipated as a design basis. This design approach has been developed based on historical precedent, the observation of steel building performance in past earthquakes, limited research that has included laboratory testing of beam-column models (albeit with mixed results), and nonlinear analytical studies.

2.2.3 Damage to Welded Steel Moment-Frame (WSMF) Construction in the 1994 Northridge, California, Earthquake

Following the apparent widespread discovery of steel frame damage in the 1994 Northridge earthquake, the City of Los Angeles enacted an ordinance requiring mandatory inspections of approximately 240 buildings located in the zones of heaviest ground shaking within the City. This ordinance required that a report be filed for each building, indicating that inspections had been performed in accordance with FEMA 267, *Interim Guidelines: Evaluation, Repair, Modification and Design of Welded Steel Moment Frame Structures*, or other suitable approach, and that repairs be made. The resulting database of reported information provides a good overview of the types of damage sustained by buildings in the Northridge earthquake, though some damaged buildings, located in the zones of the most severe ground shaking, were outside the City of Los Angeles and were not included under the ordinance.

Review of statistics obtained from a data base of the damage reported under this ordinance program indicates that the damage was less severe than had originally been perceived. Reports for approximately one third of the buildings affected by the ordinance indicated that no damage was found in the structures. Reports for another one third of the buildings indicated only that there were rejectable defects at the roots of some beam-flange-to-column-flange welds. At the time these inspections were made, there was some uncertainty as to whether such conditions were actually damage or poor quality construction, which had not been detected during the original performance of construction quality assurance, but these conditions were routinely reported as damage. More recent investigations strongly suggest that these weld root flaws are not earthquake damage, but defects from the original construction. Only one third of the total reports prepared under the Los Angeles City ordinance indicated damage other than weld root defects. Of the buildings with reported damage other than weld root defects, two-thirds had less than 10% of their connections fractured. Only 11% of all the buildings included in the ordinance had more than 10% of their connections damaged, while relatively few buildings (13% of the total) accounted for 90% of all damage other than defects at the weld roots.

The distribution of damage in these buildings points to some important potential findings. The concentration of severe damage in a relatively small percentage of the total buildings inspected would seem to indicate that in order to sustain severe damage, a steel moment-frame building must either experience very strong response to the earthquake ground motion, or, as a result of design configuration or construction quality, or both, be particularly susceptible to damage. It would seem that most steel moment-frame buildings are not particularly susceptible to severe damage under ground shaking of Modified Mercalli Intensity VII or less.

Although initial reports following the 1994 Northridge earthquake indicated that more than 100 buildings had sustained severe damage, in many cases this reported damage was limited to discontinuities and defects at the root of the complete joint penetration (CJP) welds between the beam bottom flange and the column flange. As previously noted, there is strong evidence to suggest that most such conditions are not damage at all, but rather, pre-existing construction defects that were not detected during the original construction quality assurance program. Subsequent research in other buildings and cities suggests that the presence of such defects is widespread and generally present in the population of welded steel moment-frame (WSMF) buildings constructed in the United States prior to the Northridge earthquake.

Notwithstanding the above comments, a number of buildings did experience brittle fracture damage in their beam-column connections. The amount of damage sustained by buildings was generally related to the severity of ground shaking experienced at the building site as well as the severity of response of the structure to the ground shaking, although this second factor was not necessarily measured during the earthquake. However, the presence of construction defects in the welded joints was also a significant factor in the initiation of fracture damage. Joints with severe defects at the weld roots were more susceptible to fracture initiation than joints without such defects. Since the distribution of joints with defects in an existing structure is somewhat random, this tends to minimize the effectiveness of structural analysis in predicting the exact locations where damage is likely to occur under ground shaking. However, probabilistic methods based on structural analysis can be successful in indicating the general likelihood of damage, given certain levels of ground shaking. Therefore, the evaluation and design criteria contained in these *Recommended Criteria* are based on such probabilistic approaches.

Commentary: Detailed information on the types of damage discovered in various WSMF buildings following past earthquakes may be found in a companion report, FEMA-355E - State of the Art Report on Past Performance of Steel Moment-Frame Buildings in Earthquakes.

2.2.4 Damage to Welded Steel Moment-Frame (WSMF) Construction in Other Earthquakes

Following the discovery of unanticipated damage to WSMF construction in the 1994 Northridge earthquake, engineers and building officials became concerned that similar, but as yet undetected damage, had occurred in WSMF buildings that had been affected by other earthquakes, such as the 1989 Loma Prieta earthquake in the San Francisco Bay Area. A concerted effort was undertaken by this project to determine the amount and extent of earthquake damage resulting from this and other recent earthquakes. Specifically, available WSMF damage information was gathered from the 1989 Loma Prieta, 1992 Landers, and 1992 Big Bear events. Unfortunately, since no mandatory inspection programs of WSMF buildings were enacted following these other earthquakes, the available data is not complete. It was, however, possible to confirm that six buildings in the San Francisco Bay Area sustained connection fractures in the Loma Prieta earthquake and one building in Big Bear, California sustained connection fractures as a result of the 1992 events. This confirms that the damage experienced in the 1994 Northridge earthquake was not a result either of unique ground shaking characteristics produced by that earthquake or of design and construction practices unique to the Los Angeles region. Further details of these investigations may be found in *FEMA-355E*.

One year to the day following the Northridge earthquake, on January 17, 1995, a magnitude 6.9 earthquake occurred near Kobe, Japan. Kobe is a large city with a population of about 1.5 million and had many WSMF structures in its building stock. These structures ranged from relatively small and low-rise buildings constructed in the 1950s and 1960s to modern high-rise structures constructed within the preceding 10 years. Design and construction practice in Japan is significantly different from common practice in the United States. Many of the smaller Japanese steel moment-frame (WSMF) structures employ cold-formed, tubular steel columns, with the beams, rather than columns, running continuously through the moment-resisting connections. In a detailed study of the damage sustained by 630 modern steel buildings in the heavily shaken area, the Building Research Institute of Japan determined that approximately one third experienced no significant damage, one third relatively minor damage, and the remaining third severe damage, including partial or total collapse of approximately half of the buildings in this remaining third (*FEMA-355E*). Just as in the United States, the Japanese believed that this damage was serious enough to warrant investment in a large program of research and development to determine the cause of the poor performance of WSMF buildings and to develop new techniques for design and construction of more reliable WSMF buildings.

2.2.5 Post-Northridge Earthquake Construction Practice

Investigation of the damage that occurred in the 1994 Northridge earthquake revealed a number of factors believed to have contributed to the poor performance of WSMF structures. These included the following:

- It was common practice to use large framing members even in relatively small buildings. Initial testing of WSMF connections, conducted in the 1960s and 1970s, utilized assemblies that employed small-sized elements, typically W18 beams and light W12 and W14 column sections. Typical buildings damaged by the Northridge earthquake employed W30 or larger beams connected to heavy W14 columns. It appears that size plays a significant role in the behavior of WSMF connections and that details that behave well for connections of small sections do not necessarily behave as well for larger sections.
- Typical detailing practice prior to the Northridge earthquake relied on the development of large inelastic behavior within the beam-column connections. This was the case even though one of the basic rules of detailing structures for superior seismic performance is to design connections of elements such that the connection is stronger than the elements themselves, so that any inelastic behavior occurs within the element and not the connection. There are several reasons for this rule. The strength and ductility of any connection is highly dependent on the quality of the workmanship employed. Connections, being relatively limited in size, must undergo extreme local yielding if they are to provide significant global ductility. The basic fabrication process for connections, employing cutting, welding, and bolting, tends to induce a complex series of effects on both the residual stress state and metallurgy of the connected parts that is often difficult to predict. Despite these common axioms of earthquake-resistant design the connections were called on for large inelastic behavior.
- Welding procedures commonly employed in the erection of WSMF buildings resulted in deposition of low-notch-toughness weld metal in the critical beam-flange-to-column-flange joints. This weld metal is subject to the initiation and development of unstable brittle fractures when subjected to high stress and strain demands and used in situations with significant geometric stress risers, or notches.
- Welding practice in many of the damaged structures was found to be sub-standard, despite the fact that quality assurance measures had been specified in the construction documents and

that construction inspectors had signed documents indicating that mandatory inspections had been performed. Damaged welds commonly displayed inadequate fusion at the root of the welds as well as substantial slag inclusions and porosity. These defects resulted in ready crack initiators that enabled brittle fractures to initiate in the low-toughness weld metals.

- Detailing practice for welded joints inherently resulted in the presence of fracture-initiators. This includes failure to remove weld backing and runoff tabs from completed joints. These joint accessories often contain or obscure the presence of substandard welds. In addition, they introduce geometric conditions that are notch-like and can serve as fracture initiators.
- The presence of low-notch-toughness metal in the fillet region of some structural shapes can contribute to early fractures. The metallurgy of the material in the fillet or "k-area" region of a rolled shape often has lower notch toughness properties than material in other locations of the section due to a number of shape production factors including a relatively prolonged cooling period for this area, as well as significant cold working during shape straightening. While not normally a problem, the combined presence of weld access holes through this region at the beam-column connection and large induced stresses from buckling and yielding of the beam flanges under inelastic frame action can result in initiation of fractures in this region. These problems are made more severe by improperly cut weld access holes, which can result in sharp notches and crack initiation points. This was not a common problem in the Northridge earthquake because most connections that experienced damage did so because of other, more significant vulnerabilities. However, some of the damage that occurred to Japanese structures in the Kobe earthquake was apparently the result of these problems.
- In the 1980s, some engineers came to believe that shear yielding of the panel zones in a beam-column connection, as opposed to flexural hinging of the beam, was a more benign and desirable way to accommodate frame inelastic behavior. In response to this, in the mid-1980s the building code was modified to include provisions that allowed the design of frames with weak panel zones. Contrary to the belief that panel-zone yielding is beneficial and desirable, excessive yielding actually produces large secondary stresses at the beam-flange-to-column-flange joint, which can exacerbate the initiation of fractures.
- The yield strength of structural shape material had become highly variable. In the 1980s and 1990s, the steel production industry in the United States underwent a major realignment with new mills coming on-line and replacing older mills. Although there had always been significant variation in the mechanical properties of structural steel material, the introduction of material produced by these newer mills resulted in significant additional variation. The newer mills used scrap-based steel production, which tends to produce higher-strength material than did the older mills. In fact, much of the A36 material produced by these newer mills also met the strength requirements for the higher strength A572, Grade 50 specification. Many designers had traditionally specified A572 material for columns and A36 material for beams, in order to obtain structures economically with weak beams and strong columns. The introduction of higher strength A36 material into the market effectively negated the intent of this specification practice and often resulted in frame assemblies in which the beams were stronger than the columns or panel zones were weaker than intended, relative to the beam strength. These combined effects resulted in greater strength demands on welded joints.

- The typical steel moment-frame beam-column connection inherently incorporated a number of stress concentrations. Although design calculations of connection capacity assume that stresses are uniformly distributed across beam flanges and that flexural stresses are carried primarily by the flanges while shear stresses are carried primarily by the web, in reality, the flange also carries significant local bending and shear stress and stresses are not uniformly distributed within the flange elements. The result of this is that large stress and strain demands occur at various locations, including the center of the weld root of the welded beam-flange-to-column-flange joint. This exacerbates the tendency of the weld defects, which are common in this region, to initiate brittle fractures in the low-notch-toughness metal. This effect is further exacerbated by the fact that the material at the center of the beam-flange-to-column-flange joint is under high tri-axial restraint. Under these conditions the material cannot yield, but rather will respond to stress in an elastic manner until the ultimate tensile strength is exceeded, at which time it initiates fracture. This problem is most severe when heavy sections are used, as the thicker material provides greater restraint.

Following the discovery of the susceptibility of typical pre-Northridge connections to fracture damage, an emergency change to the *Uniform Building Code* was adopted by the International Conference of Building Officials, removing the prequalified status of the typical bolted-web, welded-flange moment connection previously prescribed by the code and substituting in its place requirements that each connection design be qualified by a program of prototype laboratory testing. In 1994, the University of Texas at Austin engaged in a limited program of connection testing, using funding provided by the American Institute of Steel Construction and a private institution. That testing indicated that connections reinforced with cover plates to encourage the formation of plastic behavior within the span of the beam, away from the face of the columns, could provide acceptable behavior. This detail is illustrated in Figure 2-3. During the period 1994-1996 this became the most commonly specified connection type.

In the earliest connections of this type, welding was performed with electrodes that deposited material without rated notch toughness and with a wide variety of cover plate configurations. In August, 1995, *FEMA-267* was published, providing a standardized methodology for design of these connections, and the design and fabrication of these connections became more consistent. *FEMA-267* required the use of weld filler metals with rated notch toughness, and also included information on other types of connections that were believed capable of providing acceptable performance, including haunched connections, reduced-beam-section connections, vertical rib plate connections, side plate connections and slotted web connections. The recommendations contained in *FEMA-267* were based on preliminary research and were of an interim nature. While it is expected that frames constructed with connections designed using the *FEMA-267* guidelines are more resistant to connection fractures than earlier frames, it should not be assumed that they are completely free of potential for such damage.

Subsequent to the publication of *FEMA-267*, numerous other connection types have been developed and tested. For the upgrade of existing buildings, solutions utilizing connection modifications are discussed in Chapter 5 of these *Recommended Criteria* and supporting information is presented in Chapter 6, Connection Qualification.

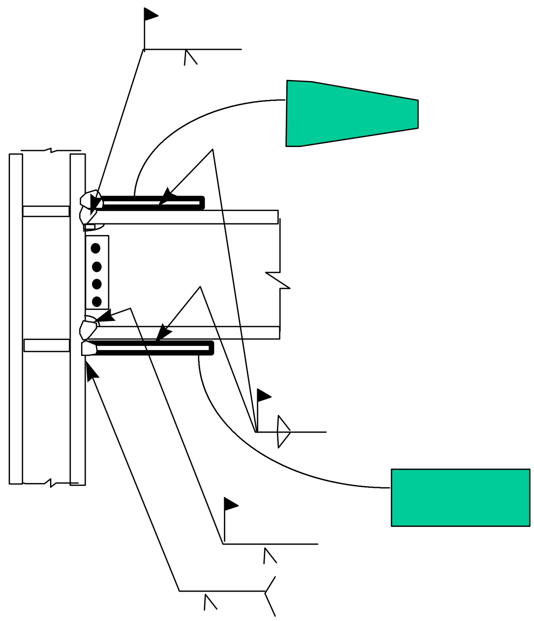

Figure 2-3 Typical Cover Plate Connection

2.3 Typical Pre-Northridge Connection Damage

Following the 1994 Northridge earthquake, damage to elements of welded steel moment frames (WSMF) was generally categorized according to a system published in *FEMA-267*. Under this system, damage is categorized as belonging to the weld (W), girder (G), column (C), panel zone (P), or shear tab (S) categories. Damage at a connection may be confined to one category or may include multiple types. The damaged WSMF may also exhibit global effects, such as permanent interstory drifts. The components of a typical pre-Northridge connection are shown in Figure 2-4.

Observation of damage sustained by buildings in the Northridge earthquake indicates that in many cases brittle fractures initiated within the connections at very low levels of plastic demand, and in some cases, while the structures remained elastic. Typically, but not always, fractures initiated at the complete joint penetration (CJP) weld between the beam bottom flange and column flange as shown in Figure 1-2. Once initiated, these fractures progressed along a number of different paths, depending on the individual joint conditions.

In some cases, the fracture progressed completely through the thickness of the weld, and if fire protective finishes were removed, the fracture was evident as a crack through exposed faces of the weld, or the metal just behind the weld (Figure 1-3a). Other fracture patterns also developed. In some cases, the fracture developed into a through-thickness failure of the column flange material behind the CJP weld (Figure 1-3b). In these cases, a portion of the column flange remained bonded to the beam flange, but pulled free from the remainder of the column. This fracture pattern has sometimes been termed a “divot” or “nugget” failure.

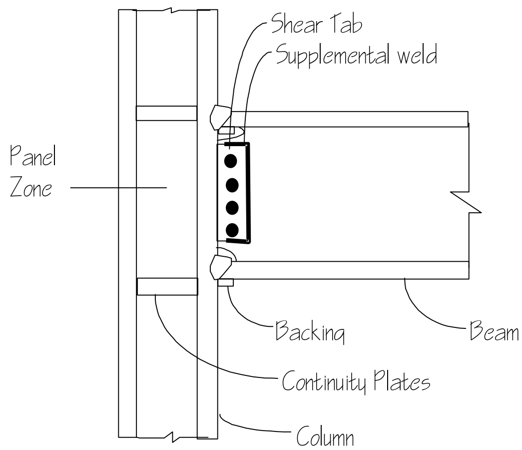

Figure 2-4 Components of Moment Connection

A number of fractures progressed completely through the column flange, along a near-horizontal plane that aligns approximately with the beam lower flange (Figure 1-4a). In some cases, these fractures extended into the column web and progressed across the panel zone Figure (1-4b). Investigators have reported some instances where columns fractured entirely across the section.

Once such fractures have occurred, the beam-column connection has experienced a significant loss of flexural rigidity and strength. Residual flexural strength and rigidity must be developed through a couple consisting of forces transmitted through the remaining flange connection and the web bolts. However, in providing this residual strength and stiffness, the beam shear connections can themselves be subject to failures, consisting of fracturing of the welds of the shear plate to the column, fracturing of supplemental welds to the beam web or fracturing through the weak section of shear plate aligning with the bolt holes (Figure 1-5).

Despite the obvious local strength impairment resulting from these fractures, many damaged buildings did not display overt signs of structural damage, such as permanent drifts, or extreme damage to architectural elements. The following sections detail typical damage types, using the system for categorizing damage recommended in *FEMA-352 – Recommended Postearthquake Evaluation and Repair Criteria for Welded Steel Moment-Frame Buildings* for postearthquake damage assessment.

2.3.1 Girder Damage

Girder damage may consist of yielding, buckling or fracturing of the flanges of girders at or near the girder-column connection. Eight separate types are defined in Table 2-1. Figure 2-5 illustrates these various types of damage.

Minor yielding of girder flanges (type G2) is the least significant type of girder damage. It is often difficult to detect and may be exhibited only by local flaking of mill scale and the formation of characteristic visible lines in the material, running across the flange. If a finish, or fireproofing has been removed by scraping, the detection of this type of damage is difficult.

Girder flange yielding, without local buckling or fracture, results in negligible degradation of structural strength.

Table 2-1 Types of Girder Damage

Type	Description
G1	Buckled flange (top or bottom)
G2	Yielded flange (top or bottom)
G3	Flange fracture in heat affected zone (HAZ) (top or bottom)
G4	Flange fracture outside heat affected zone (HAZ) (top or bottom)
G5	Flange fracture top and bottom (not used)
G6	Yielding or buckling of web
G7	Fracture of web
G8	Lateral torsional buckling of section

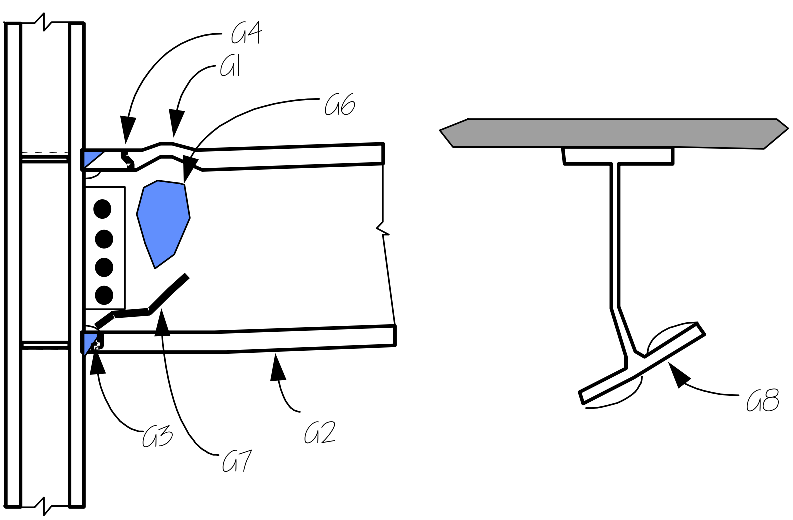

Figure 2-5 Types of Girder Damage

Girder flange buckling (type G1) can result in a significant loss of girder plastic strength, particularly when accompanied by girder web buckling (type G-6). For compact sections, this strength loss occurs gradually, and increases with the number of inelastic cycles and the extent of the inelastic excursion. Following the initial onset of buckling, additional buckling will often occur at lower load levels and result in further reductions in strength, from the levels of previous cycles. The localized secondary stresses which occur in the girder flanges due to the buckling can result in initiation of flange fracture damage (G4) if the frame is subjected to a large number of cycles. Such fractures typically progress slowly, over repeated cycles, and grow in a ductile manner. Once this type of damage initiates, the girder flange will begin to lose tensile capacity under continued or reversed loading, although it may retain some capacity in compression.

In structures with low-toughness welds, girder flange cracking within the heat-affected zone (type G3) can occur as an extension of brittle fractures that initiate in the weld root. This is particularly likely to occur at connections in which improper welding procedures were followed, resulting in a brittle heat-affected zone. However, these fractures can also occur in connections with tough welded joints (made following appropriate procedures), as a result of low-cycle fatigue, exacerbated by the high stress concentrations that occur at the toe of the weld access hole, in unreinforced beam-column connections. Like the visually similar type G4 damage, which can also result from low-cycle fatigue conditions at the toe of the weld access hole, it results in a complete loss of flange tensile capacity, and consequently significant reduction in the contribution to frame lateral strength and stiffness from the connection.

In the 1994 Northridge earthquake, girder damage was most commonly detected at the bottom flanges, although some instances of top flange failure were also reported. There are several reasons for this. First the composite action induced by the presence of a floor slab at the girder top flange tends to shift the neutral axis of the beam towards the top flange. This results in larger tensile deformation demands on the bottom flange than on the top. In addition, the presence of the slab tends to reduce the chance of local buckling of the top flange. The bottom flange, however, being less restrained can experience buckling relatively easily.

There are a number of other factors that could lead to a greater incidence of bottom flange fractures. The location of the weld root and backing are among the most important of these. At the bottom flange joint, the backing is located at the extreme tension fiber, while at the top flange it is located at a point of lesser stress and strain demand for three reasons: (1) it is located on the inside face of the flange, (2) the local bending introduced in the flanges as a result of panel zone shear deformations, and (3) because of the presence of the floor slab. Therefore, any notch effects created by root defects and backing are more severe at the bottom flange. Another important factor is that welders can typically make the complete joint penetration groove weld at the girder top flange without obstruction, while the bottom flange weld must be made with the restriction induced by the girder web. Also the welder typically has better access to the top flange joint. Thus, top flange welds tend to be of higher quality, and have fewer stress risers, which can initiate fracture. Finally, studies have shown that inspection of the top flange weld is more likely to detect defects accurately than inspection at the bottom flange, contributing to the better quality likely to occur in top flange welds.

2.3.2 Column Flange Damage

Seven types of column flange damage are defined in Table 2-2 and illustrated in Figure 2-6. Column damage typically results in degradation of a structure’s gravity-load-carrying strength as well as lateral-load resistance.

Table 2-2 Types of Column Damage

Type	Description
C1	Incipient flange crack
C2	Flange tear-out or divot
C3	Full or partial flange crack outside heat-affected zone
C4	Full or partial flange crack in heat-affected zone
C5	Lamellar flange tearing
C6	Buckled flange
C7	Column splice failure

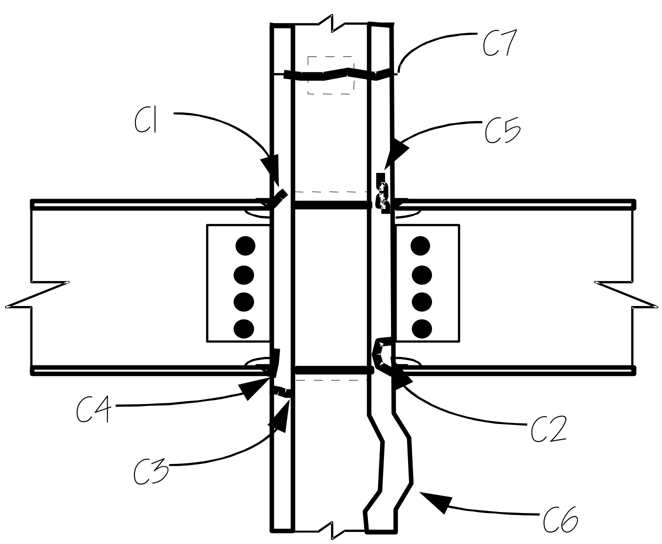

Figure 2-6 Types of Column Damage

Column flange damage includes types C1 through C7. Type C1 damage consists of a small crack within the column flange thickness, typically at the location of the adjoining girder flange. C1 damage does not go through the thickness of the column flange and can be detected only by nondestructive testing. Type C2 damage is an extension of type C1, in which a curved failure surface extends from an initiation point, usually at the root of the girder-to-column-flange weld, and extends longitudinally along the column flange. In some cases this curved failure surface may emerge on the same face of the column flange as the one where it initiated. When this occurs, a characteristic nugget or divot can be withdrawn from the flange. Types C3 and C4 fractures extend through the thickness of the column flange and may extend into the panel zone. Type C5 damage is characterized by a step-shaped failure surface within the thickness of the column flange and aligned parallel to it. This damage is often detectable only with the use of nondestructive testing.

Type C1 damage does not result in an immediate large strength loss in the column; however, such small fractures can easily progress into more serious types of damage if subjected to additional large tensile loading by aftershocks or future earthquakes. Type C2 damage results in both a loss of effective attachment of the girder flange to the column for tensile demands and a significant reduction in available column flange area for resistance of axial and flexural demands. Type C3 and C4 damage result in a loss of column flange tensile capacity and, under additional loading, can progress into other types of damage.

Type C5, lamellar tearing damage, may occur as a result of non-metallic inclusions within the column flange, particularly in older steels, when, prior to rolling, segregation of alloy inclusions was not controlled as well as in modern steels. The potential for this type of fracture under conditions of high restraint and large through-thickness tensile demands has been known for a number of years and has sometimes been identified as a contributing mechanism for type C2 column flange through-thickness failures. No lamellar tearing failures were identified after the Northridge earthquake.

Type C6 damage consists of local buckling of the column flange, adjacent to the beam-column connection. While such damage was not actually observed in buildings following the 1994 Northridge earthquake, it can be anticipated at locations where plastic hinges form in the columns. Buckling of beam flanges has been observed in the laboratory at interstory drift demands in excess of 0.02 radians. Column sections are usually more compact than beams and therefore are less prone to local buckling. Type C6 damage may occur, however, in buildings with strong-beam-weak-column systems and at the bases of columns in any building when large interstory drifts have occurred.

Type C7 damage, fracturing of welded column splices, also was not observed following the Northridge earthquake. However, the partial penetration groove welds commonly used in these splices are susceptible to fracture when subjected to large tensile loads. Large tensile loads can occur on a column splice as a result of global overturning effects, or as a result of large flexural demands in the column.

2.3.3 Weld Damage, Defects, and Discontinuities

Three types of weld damage are defined in Table 2-3 and illustrated in Figure 2-7. All apply to the complete joint penetration welds between the girder flanges and the column flanges.

Type W2 fractures extend completely through the thickness of the weld metal and can be detected by either Magnetic Particle Testing (MT) or Visual Inspection (VI) techniques. Type W3 and W4 fractures occur at the zone of fusion between the weld filler metal and base material of the girder and column flanges, respectively. All three types of damage result in a loss of tensile capacity of the girder-flange-to-column-flange joint.

Table 2-3 Types of Weld Damage, Defects and Discontinuities

Type	Description
W2	Crack through weld metal thickness
W3	Fracture at column interface
W4	Fracture at girder flange interface

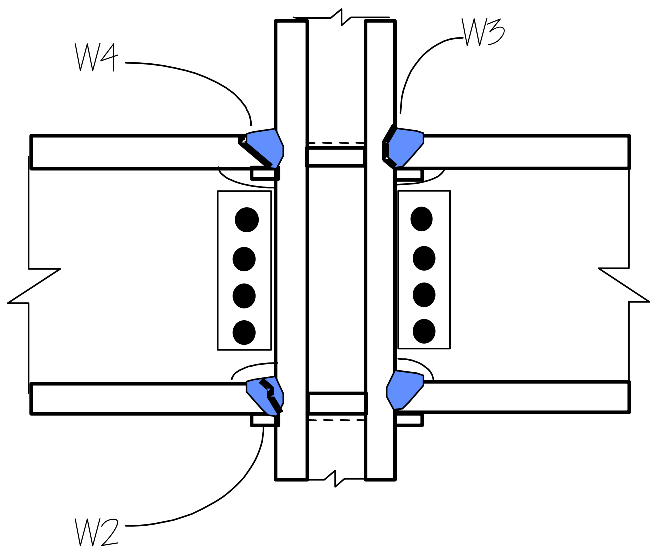

Figure 2-7 Types of Weld Damage

In addition to the W2, W3, and W4 types of damage indicated in Table 2-3 and Figure 2-7, the damage classification system presented in *FEMA-267* included conditions at the root of the complete joint penetration weld that did not propagate through the weld nor into the surrounding base metal, and could be detected only by removal of the weld backing or through the use of nondestructive testing. These conditions were termed types W1a, W1b, and W5.

As defined in *FEMA-267*, type W5 consisted of small discontinuities at the root of the weld, which, if discovered as part of a construction quality control program for new construction would not be rejectable under the *AWS D1.1* provisions. *FEMA-267* recognized that W5 conditions were likely to be the result of acceptable flaws introduced during the initial building construction, but included this classification so that it could be reported in the event that it was detected in the course of the ultrasonic testing that *FEMA-267* required. There was no requirement to repair such conditions.

Type W1a and W1b conditions, as contained in *FEMA-267* consisted of discontinuities, defects and cracks at the root of the weld that would be rejectable under the *AWS D1.1* provisions. W1a and W1b were distinguished from each other only by the size of the condition. Neither condition could be detected by visual inspection unless weld backing was removed, which, in the case of W1a conditions, would also result in removal of the original flaw or defect. At the time *FEMA-267* was published, there was considerable controversy as to whether or not the various types of W1 conditions were actually damage or just previously undetected flaws introduced during the original construction. Research conducted since publication of *FEMA-267* strongly supports the position that most, if not all, W1 damage consists of pre-existing defects, rather than earthquake damage.

2.3.4 Shear Tab Damage

Six types of damage to girder-web-to-column-flange shear tabs are defined in Table 2-4 and illustrated in Figure 2-8. Severe damage to shear tabs is often an indication that other damage has occurred to the connection, i.e., to the column, girder, panel zone, or weld.

Table 2-4 Types of Shear Tab Damage

Type	Description
S1	Partial crack at weld to column
S2	Fracture of supplemental weld
S3	Fracture through tab at bolts or severe distortion
S4	Yielding or buckling of tab
S5	Loose, damaged or missing bolts
S6	Full length fracture of weld to column

Shear tab damage should always be considered significant, as failure of a shear tab connection can lead to loss of gravity-load-carrying capacity for the girder, and potentially partial collapse of the supported floor. Severe shear tab damage typically does not occur unless other significant damage has occurred at the connection. If the girder flange joints and adjacent base metal are sound, they prevent significant differential rotations from occurring between the column and girder. This protects the shear tab from damage, unless excessively large shear demands are experienced. If these excessive shear demands do occur, than failure of the shear tab is likely to trigger distress in the welded joints of the girder flanges.

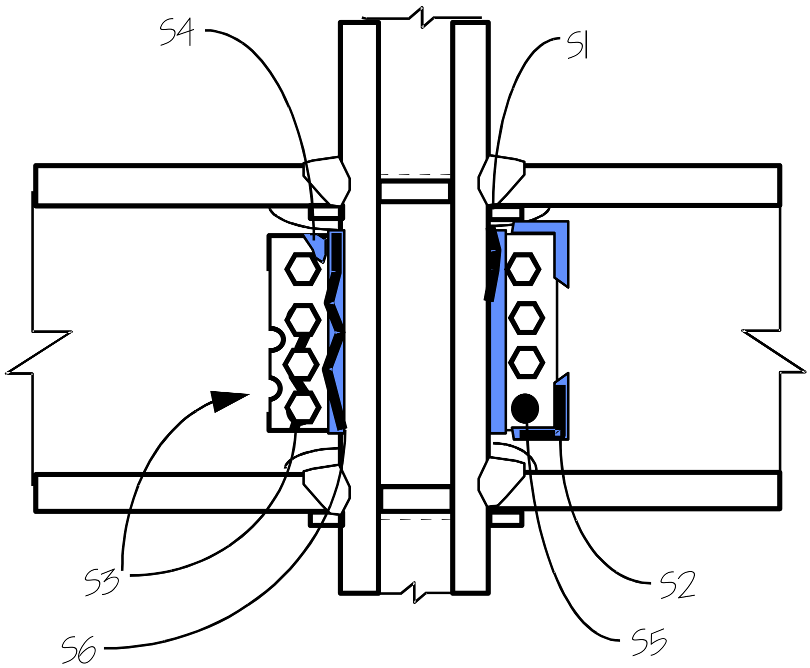

Figure 2-8 Types of Shear Tab Damage

2.3.5 Panel Zone Damage

Nine types of damage to the column web panel zone and adjacent elements are defined in Table 2-5 and illustrated in Figure 2-9. This class of damage can be among the most difficult to detect since elements of the panel zone may be obscured by beams framing into the weak axis of the column.

Fractures in the welds of continuity plates to columns (type P2), or damage consisting of fracturing, yielding, or buckling of the continuity plates themselves (type P1) may be of relatively little consequence to the structure, so long as the fracture does not extend into the column material itself. Fracture of doubler plate welds (type P4) is more significant in that this results in a loss of effectiveness of the doubler plate and the fractures may propagate into the column material.

Although shear yielding of the panel zone (type P3) is not by itself undesirable, under large deformations such shear yielding can result in kinking of the column flanges and can induce large secondary stresses in the girder-flange-to-column-flange connection. In testing conducted at the University of California at Berkeley, excessive deformation of the column panel zone was identified as a contributing cause to the initiation of type W2 fractures at the top girder flange. It is reasonable to expect that such damage could also be initiated in real buildings, under certain circumstances.

Fractures extending into the column web panel zone (types P5, P6 and P7) have the potential, under additional loading, to grow and become type P9, a complete disconnection of the upper half of the column within the panel zone from the lower half, and are therefore potentially as severe as column splice failures. When such damage has occurred, the column has lost all tensile capacity and its ability to transfer shear is severely limited. Such damage results in a total loss of reliable seismic capacity. It appears that such damage is most likely to occur in connections that are subject to column tensile loads, or in connections in which beam yield strength exceeds the yield strength of the column material.

Table 2-5 Types of Panel Zone Damage

Type	Description
P1	Fracture, buckle or yield of continuity plate
P2	Fracture in continuity plate welds
P3	Yielding or ductile deformation of web
P4	Fracture of doubler plate welds
P5	Partial depth fracture in doubler plate
P6	Partial depth fracture in web
P7	Full or near full depth fracture in web or doubler
P8	Web buckling
P9	Severed column

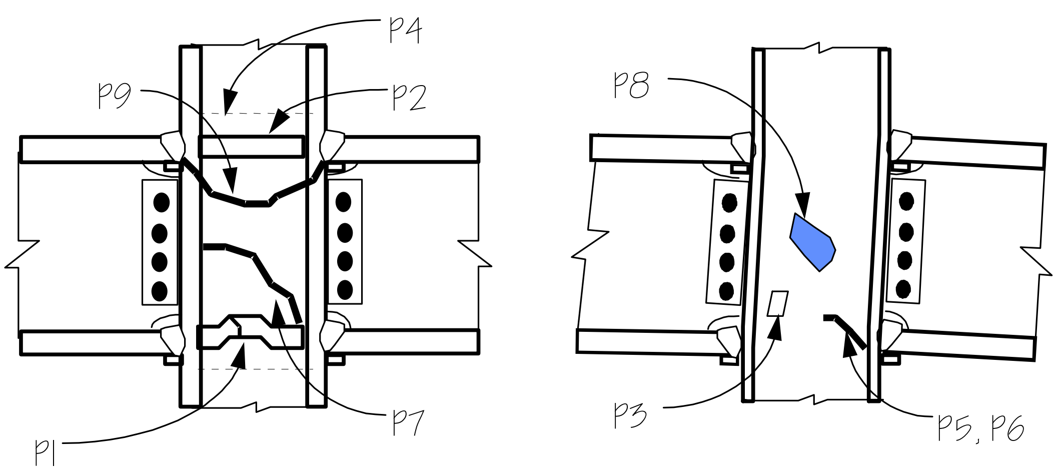

Figure 2-9 Types of Panel Zone Damage

Panel-zone web buckling (type P8) may result in rapid loss of shear stiffness of the panel zone with potential total loss of reliable seismic capacity. Such buckling is unlikely to occur in connections which are stiffened by the presence of a vertical shear tab for support of a beam framing into the column's minor axis.

2.3.6 Other Damage

In addition to the types of damage discussed in the previous sections, other types of structural damage may also be found in steel moment-frame buildings. Other framing elements which may experience damage include: (1) column base plates, beams, columns, and their connections that were not intended in the original design to participate in lateral force resistance, and (2) floor and roof diaphragms. In addition, large permanent interstory drifts may develop in the structures. Based on observations of structures affected by the 1994 Northridge earthquake, such damage is unlikely unless extensive damage has also occurred to the lateral-force-resisting system.

2.4 Evaluation Procedures

This document provides recommendations for performing several types of evaluation of the probable performance of existing steel moment-frame buildings in future earthquakes, as outlined below:

- **Performance Evaluation.** The purpose of performance evaluation is to permit estimation of a level of confidence that a structure will be able to achieve a desired performance objective (i.e., have less than a given probability of experiencing damage in excess of one or more defined limit states). In these *Recommended Criteria*, building damage is characterized in terms of two performance levels. Section 3.2.2 provides definitions of these performance levels. Once a performance objective for a building has been selected, a performance evaluation can be performed in accordance with Section 3.3 to determine a level of confidence with regard to the structure's ability to meet this performance objective. The level of confidence that can be attained with regard to the ability of a building to meet a desired performance objective is dependent on the amount of information that is available with regard to the building's configuration and construction, and the rigor of the analytical methods used in the evaluation. The performance evaluation procedures contained in Section 3.3 include simple methods for the quantification of uncertainty and confidence with regard to performance prediction of regular, well-behaved structures. More detailed methods, that permit more certain evaluation of performance capability, and which must be used for evaluation of irregular buildings are contained in Appendix A. Procedures and information regarding material properties and condition assessments to be utilized in support of the performance evaluation are presented in Section 2.5.

  Commentary: In recent years, a series of standardized building performance evaluation methodologies, including ATC-14, FEMA-154, FEMA-178 and most recently FEMA-310, have been developed. These methodologies were developed to provide the engineering community with consistent yet economical methods of determining the probable performance of different types of buildings when subjected to specific earthquake ground shaking levels. Evaluations performed in accordance with these methodologies generally consist of responding to a series of evaluation statements, intended to identify the presence of certain common *vulnerabilities, such as soft stories, weak stories, and discontinuous lateral-force-resisting systems that have been frequently observed to result in poor building performance in the past. These methodologies also commonly employ a series of analytical evaluations that include approximate evaluations of building strength and stiffness.*

While these methodologies provide good screening criteria to identify those buildings that have obvious vulnerabilities, and also serve to identify those buildings that have outstanding seismic performance characteristics, the approximate analytical procedures employed in these methods inherently incorporate so much uncertainty as to make them relatively ineffective for quantifying building performance.

Nevertheless, it is recommended that FEMA-310 be performed as a first step in the analytical evaluation of a building's probable seismic performance. Such an evaluation will provide the engineer with a basic understanding of potential critical flaws in the building configuration and provide a basis for a more detailed analytical evaluation of the building's performance, under the procedures of these Recommended Criteria.

- **Loss Evaluation**. The purpose of a loss evaluation is to determine the probable repair costs for a structure (or class of structures), if it is subjected to an earthquake hazard of defined intensity. In most loss-estimation methodologies, repair costs are expressed as a percentage of the building replacement cost. Loss-estimation evaluations sometimes include estimates of potential interruption of building occupancy as well as repair cost. Two approaches to loss estimation are provided herein: a rapid loss-estimation methodology and a detailed loss-estimation method. Rapid loss estimation, described in Chapter 4, can be quickly performed using basic data on the building's construction characteristics and specification of the intensity of ground shaking for which the loss evaluation is being performed. Detailed loss estimation requires an analytical evaluation of the building and estimation of the ground shaking response accelerations at which different damage states are likely to be exceeded. Appendix B provides information on detailed loss-estimation methods that are compatible with *HAZUS*, FEMA's nationally applicable earthquake-loss-estimation model.

Commentary: The rapid loss evaluation methodology is an approach similar to that taken in ATC-13 (ATC, 1985), in which the probability of experiencing a certain loss is related to the intensity of ground shaking experienced at the site, measured by the Modified Mercalli Intensity (MMI). Such methodologies were originally developed to estimate the probable distribution of losses for broad classes or populations of buildings. These methodologies are generally based on either actuarial statistics of the actual losses experienced by populations of buildings in past earthquakes, or on statistics related to expert opinion on the probable performance of actual buildings, or both. The methods have no direct way to account for individual building structural performance characteristics such as strength, stiffness, redundancy, or regularity, and as a result, inherently incorporate a great deal of uncertainty when applied to estimation of the loss for *a specific building structure. However, in recent years, the application of these methodologies to single building loss estimation, though technically incorrect, has become common. This application is not recommended.*

The detailed loss-estimation methodology presented in Appendix B provides for the direct consideration of structural characteristics, important to building performance, in the loss-evaluation process. In this methodology, structural analyses of the building structure are performed to characterize the probable response of the building to ground motion. Statistical data are then used to relate building response to damage and loss, at defined levels of uncertainty. The detailed loss-estimation methodology is recommended for applications in which it is desired to estimate the probable losses for a single building, as opposed to populations of buildings. It is particularly recommended as a design verification methodology for those cases when it is desired to upgrade a building to protect against future economic loss.

2.5 Material Properties and Condition Assessments

In order to perform a meaningful evaluation of either type, it is necessary to understand the structure's basic configuration, its condition, and certain basic material properties. The extent of the necessary knowledge depends on the type of evaluation and the level of certainty desired for the conclusions drawn from the evaluation. Original construction documents, including the drawings and specifications will provide sufficient data for the evaluation of most steel moment-frame buildings, so long as the building was actually constructed in accordance with these documents. As a minimum, the evaluation should include at least one visit to the building site to determine its overall condition and to confirm that available record documents are reasonably representative of the actual construction. If no construction documents are available, then extensive field surveys may be required to define the structure's configuration, including the locations of frames, the sizes of framing elements and connection details, as well as the materials of construction.

2.5.1 Material Properties

The primary material properties required to perform analytical evaluations of a steel moment-frame building include the following:

- yield strength, ultimate tensile strength and modulus of elasticity of steel for the columns in the moment frames,
- yield strength, ultimate tensile strength and modulus of elasticity of steel for the beams in the moment frames,
- ultimate tensile strength and notch toughness of the weld metal in the moment-resisting connections, and
- yield and ultimate tensile strength of bolts in the moment-resisting connections.

Although structural steel is an engineered material, there can be significant variability in the properties of the steel in a building, even if all of the members and connection elements conform to the same specifications and grades of material. Exhaustive programs of material testing to quantify the physical and chemical properties of individual beams, columns, bolts, and welds are not justified and should typically not be performed. It is only necessary to characterize the properties of material in a structure on the basis of the likely statistical distributions of the properties noted above, with mean values and coefficients of variation. Knowledge of the material specification and grade that a structural element conforms to, and its approximate age will be sufficient to define these properties for nearly all evaluations. For rapid loss-estimation evaluations, it will not be necessary to determine material properties.

In general, analytical evaluations of global building behavior are performed using expected or mean values of the material properties (based on the likely distribution of these properties) for the different grades of material present in the structure. Expected values are denoted in these procedures with the subscript "e". Thus, the expected yield and ultimate tensile strength of steel are denoted, respectively, F_{ye} and F_{ue}. Some calculations of individual connection capacities are performed using lower-bound values of strength. Where lower-bound strength values are required, the yield and tensile strength are denoted as F_y and F_u, respectively. Lower-bound strengths are defined as the mean minus two standard deviations, based on statistical data for the particular specification and grade.

If original construction documents, including drawings and specifications, are available, and indicate in an unambiguous manner the materials of construction to be employed, it will typically not be necessary to perform materials testing in a steel moment-frame building. When material properties are not clearly indicated on the drawings and specifications, or the drawings and specifications are not available, the material grades indicated in Table 2-6 may be presumed. Alternatively, a limited program of material sample removal and testing may be conducted to confirm the likely grades of these materials.

If sampling is performed, it should take place in regions of reduced stress, such as flange tips at ends of simply supported beams, flange edges in the mid-span region of members of moment-resisting frames, and external plate edges, to minimize the effects of the reduced area. If a bolt is removed for testing, a comparable bolt should be reinstalled in its place. If coupons are removed from beams or columns, the material should either be replaced with the addition of reinforcing plate, or the area of removal should be dressed to provide smooth contours of the cutout area, without square corners or notches. Removal of a welded connection sample must be followed by repair of the connection. When sampling is performed to confirm the grades of material present in a structure, mechanical properties should be determined in the laboratory using industry standard procedures in accordance with *ASTM A-370*.

For the purpose of analytical evaluation of steel moment-frame buildings, the expected and lower bound strength of structural materials shall be taken from Table 2-7, based on the age, material specification, and grade of material.

Commentary: In general, great accuracy in the determination of the material properties of structural steel elements in steel moment-frame (WSMF) buildings is neither justified nor necessary, in order to perform reasonably reliable evaluations of building performance. The two most important parameters are the yield strengths of the beams and columns and the notch toughness of the weld metal.

Weld Filler Metal

Welding was first introduced into the building construction industry in the early 1950s. Prior to that time, most structural connection was made either by riveting or bolting. Early structural welding typically used the shielded metal arc welding (SMAW) process and "stick" filler metals with an ultimate tensile strength of 60 ksi. Although a variety of weld filler metals were available, the most commonly employed filler metal in the 1950s and early 1960s conformed to the E6012 designation. In the 1960s, as higher strength steels came on the market, there was a gradual shift to the E7024 weld filler metal, which was capable of depositing metal with a 70 ksi ultimate tensile strength. Neither of these filler metals had specific rating for notch toughness, although some welds placed with these filler metals may have considerable toughness. In the mid-1960s, contractors began to switch to the semi-automatic, flux cored arc welding (FCAW) process, which permitted more rapid deposition of weld metal and therefore, more economical construction of welded structures.

Welds in most steel moment-frame buildings constructed in the period 1964-1994 were made with the FCAW process, employing either E70T-4 or E70T-7 weld filler metal. This material generally has low notch toughness at service temperatures. Precise determination of the notch toughness of individual welds is not required in order to predict the probable poor performance of moment-resisting connections made with these materials and the typical detailing of the time. However, if weld metal with significant notch toughness (40 ft-lbs at service temperature) has been used, even connections of the type typically constructed prior to the 1994 Northridge earthquake can provide limited ductility. It is rarely possible to determine the type of weld filler metal used in a building without extraction and testing of samples. Construction drawings and specifications typically do not specify the type of weld filler metal to be employed and even when they do, contractors may make substitutions for specified materials. Welding Procedure Specifications (WPS) for a project, if available, would define the type of weld filler metal employed, but these documents are rarely available for an existing building. Given the near universal use of the FCAW process with E70T-4 or E70T-7 weld filler metal during the period 1964-1994, sampling of weld metal for buildings constructed in this period is not recommended. For buildings constructed prior to 1970, sampling and testing of weld filler metal may indicate the presence of weld with superior notch toughness, which would provide a higher level of confidence that the building would be capable of meeting desired *performance objectives. Buildings constructed prior to 1964 may conservatively be assumed to be constructed using weld filler metal with low toughness, or samples may be extracted.*

Most buildings constructed after 1996 employ weld filler metals with adequate notch toughness to provide ductile connection behavior. Sampling and testing of weld metals for buildings constructed in this period are not therefore, deemed necessary. During the period 1994-96, many different types of weld filler metal were employed in buildings. Sampling and testing of weld filler metal in buildings of this period may be advisable.

When it is deemed advisable to verify the strength and notch toughness of weld filler metals, it is recommended that at least one weld metal sample be obtained and tested for each construction type (e.g., column-splice joint, or beam-flange-to-column-flange joint). Samples should consist of both local base and weld metal, such that composite strength of the connection can be assessed. If ductility is required at or near the weld, the design professional may conservatively assume that no ductility is available in the weld, in lieu of testing.

Beams and Columns

The actual strength of beam and column elements in a steel moment-frame structure is only moderately important for the performance evaluation of such structures. The primary parameter used in these Recommended Criteria to evaluate building performance is the interstory drift induced in the building by earthquake ground shaking. Building drift is relatively insensitive to the actual yield strength of the beams and columns. However, building interstory drift can be sensitive to the relative yield strengths of beams and columns. In particular, large interstory drifts can occur in buildings with weak columns and strong beams, as such conditions permit the development of a single story mechanism in which most of the building deformation is accommodated within the single story. During the 1970s and 1980s, it was common practice in some regions for engineers to specify beams of A36 material and columns of A572, Grade 50 material in order to develop economical designs with a strong-column-weak-beam configuration. If the properties of materials employed in a steel moment-frame building are unknown, it may be conservatively assumed that the beams and columns are of the same specification and grade of material, in accordance with the default values indicated in Tables 2-6 and 2-7. However, if it can be determined that different grades of material were actually used for beams and columns, it may be possible to determine a higher level of confidence with regard to the ability of a building to meet desired performance objectives. In such cases, it may be appropriate to perform a materials sampling and testing program to confirm the material specifications for beams and columns.

When it is decided to conduct a materials testing program to confirm the *specification and grade of material used in beams and columns, it is suggested that at least two strength tensile coupons should be removed from each element type for every four floors. If it is determined from testing that more than one material grade exists, additional testing should be performed until the extent of each grade has been established.*

Bolts

Bolt specifications may be determined by reference to markings on the heads of the bolts. Where head markings are obscured, or not present, the default specifications indicated in Table 2-6 may be assumed. If a more accurate determination of bolt material is desired, a representative sample of bolts should be extracted from the building and subjected to laboratory testing to confirm the material grade.

Table 2-6 Default Material Specifications for WSMF Buildings

Element Type	Age of Construction	Default Specification
Beams and Columns	1950-1960	ASTM A7, A373
	1961-1990	ASTM A36
	1990-1998	ASTM A572, Grade 50
	1999 and later	ASTM A992
Bolts	1950-1964	ASTM A307
	1964-1999	ASTM A325
Weld Filler Metal	1950-1964	E6012, E7024 (1)
	1964-1994	E70T4 or E70T7 (2)
	1994-1999	See note 3

Notes:

1 – Prior to about 1964, field structural welding was typically performed with the Shielded Metal Arc Welding (SMAW) process using either E6012 or E7024 filler metal. Neither of these electrode classifications are rated for specific notch toughness, though some material placed using these consumables may provide as much as 40 ft-lbs or greater notch toughness at typical service temperatures. It should be noted that due to other inherent characteristics of the moment resisting connection detailing prevalent prior to the 1994 Northridge earthquake, the presence of tough filler metal does not necessarily provide for reliable ductile connection behavior.

2 – During the period 1964-1994, the Flux Cored Arc Welding (FCAW) process rapidly replaced the SMAW process for field welding in building structures. Weld filler metals typically employed for this application

conformed either to the E70T4 or E70T7 designations. Neither of these weld filler metals are rated for specific notch toughness, and both have similar mechanical properties.

3 – Following the 1994 Northridge earthquake, a wide range of weld filler metals were incorporated in WSMF construction. Most of these filler metals had minimum ultimate tensile strengths of 70ksi and minimum rated notch toughness of 20 ft-lbs at –20^{o}F. However, due to the variability of practice, particularly in the period 1994-1996, limited sampling of weld metal in structures in this era is recommended to confirm these properties.

Table 2-7 Lower Bound and Expected Material Properties for Structural Steel Shapes of Various Grades

		Yield Strength (ksi)		Tensile Strength (ksi)	
Material Specification	**Year of Construction**	**Lower Bound**	**Expected**	**Lower Bound**	**Expected**
ASTM, A7, A373	pre-1960	30	35	60	70
ASTM, A36 Group 1	1961-1990	41	51	60	70
Group 2		39	47	58	67
Group 3		36	46	58	68
Group 4		34	44	60	71
Group 5		39	47	68	80
ASTM A242, A440, A441	1960-1970				
Group 1		45	54	70	80
Group 2		41	50	67	78
Group 3		38	45	63	75
Group 4		38	45	63	75
Group 5		38	45	63	75
ASTM, A572 Group 1	1970 – 1997	47	58	62	75
Group 2		48	58	64	75
Group 3		50	57	67	77
Group 4		49	57	70	81
Group 5		50	55	79	84
A36 and Dual Grade 50	1990 – 1997				
Group 1		48	55	66	73
Group 2		48	58	67	75
Group 3		52	57	72	76
Group 4		50	54	71	76

Notes:
1. Lower bound values are mean - two standard deviations, from statistical data.
2. Expected values are mean values from statistical data.
3. For wide-flange shapes, produced prior to 1997, indicated values are representative of material extracted from the web of the section.
4. For structural plate, expected strength may be taken as 125% of the minimum specified value. Lower-bound strength should be taken as the minimum specified value.

2.5.2 Component Properties

Behavior of components, including beams and columns, is dictated by such properties as area, width-to-thickness and slenderness ratios, lateral torsional buckling resistance, and connection details. Component properties of interest are:

- original cross-sectional shape and physical dimensions,
- size and thickness of additional connected materials, including cover plates, bracing, and stiffeners,
- existing cross-sectional area, section moduli, moments of inertia, and torsional properties at critical sections,
- as-built configuration of intermediate, splice, end, and base-plate connections,
- current physical condition of base metal and connector materials, including presence of deformation.

When performing detailed evaluations and loss estimates it is necessary to conduct a structural analysis of the building's response to ground motion. Each of these properties is needed to characterize building performance in the seismic analysis. The starting point for establishing component properties should be the construction documents. Preliminary review of these documents should be performed to identify primary vertical- and lateral-load-carrying elements and systems, and their critical components and connections. In the absence of a complete set of building drawings, the design professional must obtain the necessary information on section and connection properties through a program of field investigation.

2.5.3 Condition Assessment

A condition assessment of the existing building and site conditions should be performed as part of the seismic evaluation process, regardless of the type of evaluation being performed. The goals of this assessment are:

- To examine the physical condition of primary and secondary components and the presence of any degradation.
- To verify or determine the presence and configuration of components and their connections, and the continuity of load paths between components, elements, and systems.
- To review other conditions such as neighboring buildings and the presence of nonstructural components that may significantly influence building performance.

The physical condition of existing components and elements, and their connections, must be examined for presence of degradation. Degradation may influence environmental effects (e.g., corrosion, fire damage, chemical attack) or past or current loading effects (e.g., overload, damage from past earthquakes, fatigue, fracture). The condition assessment should also examine for configuration problems observed in recent earthquakes, including effects of discontinuous components, improper welding, and poor fit-up.

Component orientation, plumbness, and physical dimensions should be confirmed during an assessment. Connections in steel components, elements, and systems require special consideration and evaluation. The load path for the system must be determined, and each connection in the load path must be evaluated. This includes diaphragm-to-component and component-to-component connections.

The condition assessment also affords an opportunity to review other conditions that may influence steel elements and systems and overall building performance. Of particular importance is the identification of other elements and components that may contribute to or impair the performance of the steel system in question, including infills, neighboring buildings, and equipment attachments. Limitations posed by existing coverings, wall and ceiling space, infills, and other conditions should also be defined such that prudent rehabilitation measures may be planned.

> *Commentary: In order to perform reliable performance assessments of buildings, it is important to have knowledge of the existing condition of the building and its components. However, the framing in most welded steel moment-frame (WSMF) buildings construction is protected from deterioration by fireproofing and other building finishes, and therefore, most WSMF buildings will remain in good condition throughout their service lives. Unless a WSMF building has been subjected to an extreme loading event, such as a fire, extreme windstorm, or strong earthquake, or the structure exhibits signs of deterioration, such as rust stains, or lack of plumb, exhaustive condition surveys of WSMF structures are not generally justified, except as required to confirm that the construction conforms to the available construction documents.*

2.5.3.1 Scope and Procedures

The scope of a condition assessment should include all primary structural elements and components involved in gravity-load and lateral-load resistance.

If coverings or other obstructions exist, indirect visual inspection through use of drilled holes and a fiberscope may be utilized. If this method is not appropriate, then local removal of covering materials may be necessary. The following guidelines should be used:

- If detailed design drawings exist, exposure of at least one different primary connection should occur for each connection type. If no deviations from the drawings exist, the sample may be considered representative. If deviations are noted, then removal of additional coverings from primary connections of that type must be done until the design professional has adequate knowledge to continue with the evaluation and rehabilitation.
- In the absence of construction drawings, the design professional should establish inspection protocols that will provide adequate knowledge of the building needed for reliable evaluation.

Physical condition of components and connectors may also dictate the use of certain destructive and nondestructive test methods. If steel elements are covered by well-bonded fireproofing materials or encased in durable concrete, it is likely that their condition will be suitable. However, local removal effort is dictated by the component and element design. It may be necessary to expose more connections because of varying designs and the critical nature of the connections.

2.5.3.2 Quantifying Results

The results of the condition assessment should be used in the preparation of building system analytical models for the evaluation of seismic performance. To aid in this effort, the results should be quantified and reduced with the following specific topics addressed:

- component section properties and dimensions,
- connection configuration and presence of any eccentricities,
- type and location of column splices, and
- interaction of nonstructural components and their involvement in lateral-load resistance.

All deviations noted between available construction records and as-built conditions should be accounted for and considered in the structural analysis.